DEPARTMENT OF THE ARMY
NEW ENGLAND DIVISION, CORPS OF ENGINEERS

STABILITY ANALYSIS OF STRUCTURES

EDWARD MACDOWELL DAM

WEST PETERBOROUGH, NEW HAMPSHIRE

REPORT

CONTRACT NO. DACW 33-74-C-0065
Line Item 3

Fay, Spofford & Thorndike, Inc.
Engineers
Boston, Massachusetts

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PART I

GENERAL

I - Section 1 - Project Criteria.

List of recent and updated stability criteria and instructions provided by the Corps of Engineers, New England Division:

Engineering Manuals:

EM 1110-2-2101 - Working Stresses for Structural Design (17 Jan. 1972).
EM 1110-2-2200 - Gravity Dam Design (25 Sept. 1958).
EM 1110-2-2400 - Structural Design of Spillways and Outlet Works (2 Nov. 1964).
EM 1110-2-2501 - Wall Design: Flood Walls (18 Jun. 1962).
EM 1110-2-2502 - Retaining Walls (25 Jan. 1965).

Engineer Technical Letters:

ETL 1110-2-184 - Gravity Dam Design (25 Feb. 1974).
ETL 1110-2-109 - Structural Design for Earthquakes (21 Oct. 1970).

Pertinent Hydraulic Data:

Hydrologic Data for Structural Stability - Analysis of Spillways

List of design computations and drawings:

(1) Analysis of Design - Revised, Feb. 1948.
(2) Analysis of Design - Appendix B, Nov. 1947.
(3) Plans for Construction of West Peterborough Dam, November, 1947.
(4) Definite Project Report on West Peterborough Reservoir, May 1945.

I - Section 2 - Description of the Dam and Operating Condition.

Edward MacDowell Dam is located on Nubanusit Brook, a tributary of the Contoocook, one-half mile upstream from the village of West Peterborough and 14 miles east of Keene, New Hampshire. The dam, completed in 1950, is rolled-earthfill with a dumped rock blanket and is 67 feet high and 1,030 feet long at Elevation 967.0. It is founded on bedrock on the west bank and river section and on glacial till on the east bank. The reservoir, which is operated for flood control purposes, is normally kept empty. Control gates in the outlet works, located near the west side of the dam, are operated to store floodwaters in the reservoir during time of flood. A concrete spillway, consisting of a low weir 100 feet in length, is located in a natural saddle on the north side of the reservoir, approximately 4 miles from the dam. The spillway approach and discharge channels are cut through earth and rock. The outlet works, incorporated into the earth embankment section, include a reinforced concrete intake structure, a 7-foot by 7-foot gate-controlled conduit and a stilling basin with gravity type guide walls, all founded on rock. Three 3-foot by 7-foot mechanically operated sluice gates are provided.

The hydrological data for structural stability, updated and furnished by the Contracting Officer, are as follows:

(a) Full Pool Condition (pool at spillway crest, minimum tail water):

Energy gradient at spillway (ft. msl)	- 946.0
Tail-water energy gradient	- 938.0

(b) Design Discharge Condition (reservoir at peak level of probable maximum flood and corresponding tail waters):

Energy gradient at spillway	- 958.2
Tail-water energy gradient	- 955.0
Tail-water surface	- 945.2

I - Section 3 - Criteria for Analysis.

The principal concrete structures and project features analyzed for stability consist of the following:

(a) Intake Structure
(b) Service Bridge Pier and Abutment
(c) Spillway
(d) Spillway Walls
(e) Stilling Basin Retaining Walls

Two members of our engineering staff visited the site on February 1, 1974 (copy of memorandum enclosed).

To check sliding resistance of structures under lateral loading, a method different from the original design calculations has been used. This is the shear-friction factor of safety formula, as outlined in the Engineer Technical Letter No. 1110-2-184 of 25 Feb. 1974. The sliding resistance is a function of the angle of internal friction and the unit shearing strength of the foundation material. Where the base of the concrete structure is embedded in rock, the passive resistance of the downstream layer of rock may be utilized in addition to the sliding resistance.

In the analysis of the MacDowell Dam structures, the shear-friction safety factor formula used includes all three contributing resistances: namely, the friction, the shearing strength, and the passive reaction where applicable.

For the spillway weir, a minimum shear-friction factor of safety of 4 is required for all conditions of loading when earthquake is not considered. When earthquake is considered, this factor of safety should exceed 2-2/3. Retaining walls on earth require a sliding factor of safety of Tan $\emptyset/1.5$.

The resistance to overturning is determined according to current criteria by the location of the resultant of vertical forces at the base. Without seismic forces, the resultant should be located within the middle third. When earthquake is considered, it is acceptable if the resultant stays within the width of the base. For retaining walls founded on rock, the resultant may be outside the middle third of the base if all other conditions are met, i.e., the foundation pressures are within allowable values and the factor of safety against sliding is sufficient.

Because the MacDowell Dam is located in Zone 2 (moderate damage), as shown on the Seismic Risk Map of the U.S., included with ETL 1110-2-109, this analysis includes seismic forces as specified for that zone with acceleration of 0.10g.

The seismic forces applied to this stability analysis, in accordance with EM 1110-2-2200 of 25 Sept. 1958, are as follows:

(a) Inertia force $P_{el} = 0.10W$, acting horizontally through the center of gravity in any direction.

(b) Increase in water pressure by Westergaard's formula, first published in 1933, and expressed in terms of horizontal force P_{e2} and moment M_e at any depth y. Factor C = 51 lbs./ft.^{3} was used throughout, assuming t = 1 sec. This factor does not change appreciably for the height of structures up to 200 feet.

(c) Dynamic earth pressure in accordance with EM 1110-2-2502 of 25 Jan. 1965, was applied at about 2/3 of the fill height. This pressure is equal to about 20 percent of static lateral earth pressure. The backfill between a sloping wall and a vertical plane through the heel was added to the wall mass for calculation of inertia force P_{e1}.

(d) For walls with water on both sides, the seismic loads should include effects of increase on one side and decrease on the other side for free water. Horizontal water pressure in the soil is similar to uplift pressure and the effects of earthquake on it will be negligible; therefore, only increase in water pressure on one side is used in the design.

Ice pressure, used where applicable is 5,000 psf x 2 feet = 10,000 pounds per linear foot of structure (refer to EM 1110-2-2200, Section 2-07).

The uplift pressure at any point under a structure is the tail-water pressure plus the pressure measured as an ordinate from tail water to the hydraulic gradient between the upstream and downstream sides. In this analysis, the uplift pressure is considered to act over 100 percent of the base area measured from the upstream edge to the downstream edge.

I - Section 4 - Evaluation of Foundation.

Reference is made to "Analysis of Design," Corps of Engineers, Boston, Massachusetts, 1947, revised 1948.

The subsurface exploration consisted of field reconnaissance, seismic sampling of the overburden and rock by means of test borings and test holes, and observation wells. Seismic exploration was conducted at seven locations at the dam site and at twenty-two locations in the spillway area. These explorations were made primarily to determine approximate depth to bedrock. A total of 74 borings were drilled, 21 of them being made in the spillway.

Subsurface investigation indicates that the overburden material at the dam site consists of sandy to silty glacial till varying in depth, shallow (10 to 20 feet) at the right abutment and at the stream channel and deep at the left abutment. Bedrock consists of a silicious schist that has been severely weathered within localized zones and is highly fractured throughout the upper few feet. The overburden material at the spillway site consists of silty glacial till. The bedrock was found to be an unweathered moderately fractured porphyritic granite outcropping and at shallow depths on the southside of spillway location but dips steeply northward. The outlet conduit and control works are founded on bedrock, silicious schist, which is severely weathered within localized zones and highly fractured throughout its upper few feet both upstream and downstream from the centerline of the dam. The design report indicates that structure excavation will assure sufficient depth of rock penetration to provide a firm though fractured bedrock foundation.

The service bridge pier and abutment are founded in the compacted, pervious fill of the dam. The report made in 1947, revised in 1948, indicates that the compacted pervious fill would come from the structure excavation. Information available indicates that the material used could vary from a well graded sand to a sand and gravel. The allowable bearing pressure will vary from 2.5 tons per square foot to 3 tons per square foot. No visual overstressing of the soil or structure has been observed since the completion of the structures, even with the actual bearing pressures computed in this analysis of up to 3 tons per square foot, which indicates that the higher limit of allowable pressures can be used.

I - Section 5 - Allowable Unit Stresses at Interface of Concrete and Rock.

Allowable stresses at the bonded surface between concrete and rock are assumed to be the same as for 3000 psi concrete or as allowable for the type of rock at the site. EM 1110-1-2101 refers to the ACI Building Code for allowable stresses in concrete with certain modifications. The following allowable stresses are used in this report:

(a) Concrete - Compressive strength $f_c' =$ 3000 psi at 28 days.

(b) Rock Description - (ETL 1110-2-184, 25 Feb. 1974):

1. Spillway site, fractured porphyritic granite, 10,000 psi average compressive strength, 1300 psi average shear strength.

2. Outlet works, fractured silicious schist, 7000 psi average compressive strength, 1000 psi average shear strength.

(c) Allowable Bearing Pressure -

1. Rock
   a. Porphyritic Granite = $80t/ft.^2$ = 1,111 psi.
   b. Silicious Schist = $30t/ft.^2$ = 417 psi.

2. Bearing on compacted pervious fill = 3t/s.f.

(d) Shear at Interface Between Rock and Concrete - 40 psi (based on ACI 318-63, Composite Concrete, allowable bond shear stress for rough and clean contact surfaces without mechanical anchorage).

(e) Coefficient of Frictional Resistance - 0.5 (based on tangent of the angle of internal friction for foundation material, $\phi = 30^o$).

These allowable unit stresses may be increased 33-1/3 percent with Group II Loadings, such as wind, ice, or earthquake (EM 1110-2-2101).

PART II

RESULTS OF THE ANALYSIS

II - Section 1 - Intake Structure.

The intake structure is part of the outlet works, all reinforced concrete structure founded on sound rock. There are three gate passages. The bulkhead slots are in front of the service gates for each gate passage. Downstream from the gates there is a transition section which narrows to the 7-foot 0-inch square, cast-in-place conduit.

The control tower, which is above the intake structure, is connected to the roadway on top of the dam by a service bridge. The total height of the tower is 66 feet, measured from the bottom of the conduit slab to the operating floor at Elevation 967.0. There is a gate house superstructure built on top of the operating floor 22 feet tall. In plan, the tower measures 28 feet by 23 feet 6 inches.

The structure was analyzed for stability at two levels: Elevation 920.0 and Elevation 901.5 (on rock). Loading cases applied are those listed in EM 1110-2-2400, Section 3-07c, entitled "Stability of Gate Structure at Upstream End." The structure was analyzed for Loading Cases I through VI (III, IV, and V not governing), and IA and IIA with seismic acceleration of 0.10g for Zone 2. Obviously noncritical loadings were eliminated by comparison during the analysis. Eight loading cases were analyzed; five for stability at the base on weak axis (perpendicular to the flow), and three for the concrete section at Elevation 920.0.

Maximum bending and shear stresses at Elevation 920.0, including seismic forces, are within allowable limits.

At the base of the intake structure, Elevation 901.5, for Loading Cases I, II, and VI, the resultant is within the middle third of the base. The factor of safety against sliding is more than 1.5 for all loadings, and the bearing pressures are within the allowable values.

For Loading Cases IA and IIA, with seismic forces included, the resultant is always within the base with minimum 72 percent of base in bearing; and the maximum bearing pressure on rock of 4.8 tons per square foot does not exceed the allowable of 1.33 x 30 = 40 tons per square foot. Minimum factor of safety against sliding, based on frictional resistance only, is 3.0. The analysis does not take into account extra stability provided by embedment of the tower base into rock.

The intake structure is stable under all of the specified loading cases and no modifications or strengthening is required.

II - Section 2 - Service Bridge Pier and Abutment.

Access to the gate house is provided by a two-span service bridge from the roadway on top of the dam. The bridge has two 36-foot 0-inch spans. The roadway is 9 feet 8 inches from curb to curb and is designed for AASHO H-10 loading. The bridge is supported at one end by the intake tower, with a center pier and the abutment providing the other two supports.

The reinforced concrete pier is 24 feet high, with a pier cap, two 2-foot square columns on a 2-foot thick by 10-foot wide wall, supported on a 5-foot 6-inch by 12-foot 0-inch concrete footing.

The abutment, also reinforced concrete, is a 12-foot high gravity section on an 8-foot 0-inch by 14-foot 8-inch concrete footing. The abutment retains fill on one side up to the roadway and is surrounded by dumped rock and compacted pervious fill on the other three sides up to 2/3 of its height. The pier and the abutment are founded on the compacted pervious fill of the dam.

Loading cases considered are as given in EM 1110-2-2400, Section 3-07c, entitled "Stability of Gate Structure at Upstream End." In the analysis, horizontal soil pressure effect for both structures was neglected because the major portion of the structure is buried into the fill. Ice force on the pier was not considered because of the rock fill surrounding the pier at Elevation 946.0. Only Loading Cases IA, IIA for the pier and IA for the abutment were analyzed.

The following loading cases were used:

IA - Dead load plus earthquake, no water.

IIA - Dead load plus water level up to the top of the spillway, Elevation 946.0, plus uplift and earthquake.

For the pier, the resultant falls out of the middle third for Loading Cases IA and IIA, but at least 63 percent of the base is in the bearing. Maximum foundation pressure of 3.8 tons per square foot is less than the allowable of 1.33 x 3.0 = 4 tons per square foot and sliding factor of safety of 3.90, based on only frictional resistance, is adequate. For the abutment, the resultant is within the middle third, foundation pressures are within the allowable, and the section has an adequate factor of safety against sliding.

The pier and the abutment are stable under the specified loading conditions.

II - Section 3 - Spillway.

The spillway is a low unreinforced concrete weir, embedded in sound granite bedrock, with a crest length of 100 feet at Elevation 946.0. Steel anchors, one-inch round, spaced approximately 5 feet on centers both ways, were provided to anchor the toe securely to the rock. The entire channel in the vicinity of the weir is excavated in rock to the desired bottom elevation. The design discharge capacity is 17,000 cfs with a surcharge of 15.0 feet. However, the design discharge level of probable maximum flood at Elevation 958.2 is only 12.2 feet above the crest of the spillway.

A typical cross section of the ogee weir is 23 feet 6 inches wide and about 10.0 feet high. It consists of three monoliths, each approximately 35 feet long.

One typical section was analyzed for stability. Loading cases in accordance with EM 1110-2-2200, Section 3.01, were applied. The following loading cases were governing:

- II - Normal operating with ice pressure.
- IV - Flood discharge.
- VI - Normal operating with earthquake.

The analysis was based on the following hydrological data:

Loading Case II - Full Pool Condition (pool at spillway crest, no tail water). Pool elevation at top of spillway crest 946.0 ft. msl. Downstream water surface assumed below the concrete weir section.

Loading Case IV - Design Discharge Condition (reservoir at peak level of probable maximum flood). Energy gradient at spillway 958.2 ft. msl, tail water energy gradient 955.0 ft. msl, water surface 945.2 ft. msl.

The critical values of the factors of safety against sliding, location of resultant, and foundation pressures for the monolith analyzed are shown in Table 1.

For this typical section, the resultant falls out of the middle third for Loading Case II but is within the base with 85 percent of the base in bearing. Ice pressure used in Loading Case II was more critical than seismic forces for Loading Case VI. The remaining stability requirements are satisfied for all of the loading cases and foundation pressures for all of the cases are within the allowable bearing pressure for the rock.

An anchorage system consisting of vertical grouted anchor rods is recommended as a remedial measure and is to be designed so that the resultant will be located within the middle third of the base. The anchors, approximately 21 feet long, are to be installed and grouted into holes drilled vertically, approximately 7 feet on center, along the crest of the spillway weir, a minimum of 12 feet into rock.

The estimated cost for the anchorage system is $16,000.

TABLE 1

SPILLWAY WEIR

Weir Section	Loading Case	Location of Resultant: In Middle Third	Location of Resultant: In Base	Percent Base In Bearing	Resistance to Sliding Factor of Safety(*)	Bearing Pressures on Rock (Tons/S.F.): Maximum	Bearing Pressures on Rock (Tons/S.F.): Mininum
A-A/49	II	No	Yes	85	11.8	0.9	-
	IV	Yes	-	-	15.2	0.3	0.16
	VI	Yes	-	-	28.4	0.4	0.36

*Factor of safety is for bond shear value of 40 psi and $\phi = 30^{\circ}$.

II - Section 4 - Spillway Walls.

The spillway walls are gravity sections founded on rock. The south wall consists of three monoliths, from 7 to 15 feet high and from 7 to 12 feet wide at the base, and is approximately 80 feet long. Two of the monoliths have a concrete lining below the gravity section and are anchored to the rock by means of one-inch round anchor bars. The lining is reinforced with 3/4-inch diameter bars both ways.

The north wall, approximately 180 feet long, has no steel anchors to rock. It consists of seven monoliths, from 15 to 24 feet high and from 8 to 19 feet wide at the base.

The spillway walls are analyzed in accordance with EM 1110-2-2502 for active earth pressures, disregarding fill in front of the wall. Remedial measures for stabilizing the walls are necessary if the resultant falls outside the middle third, or if the walls do not satisfy other given criteria. Loading cases listed below were based on criteria given in EM 1110-2-2400 for approach channel walls.

Loading Cases:

North Upstream Wall -

I - Not applicable (impervious backfill).

II - Design maximum flood with partial sudden drawdown, water level in channel to Elevation 946.0, backfill submerged up to Elevation 958.2 (probable maximum flood in reservoir).

III - Design maximum flood with sudden rise in reservoir, water level in channel to Elevation 958.2, and backfill submerged up to Elevation 958.2.

IIA - II with earthquake.

North Downstream Wall -

I - Channel empty, no water in backfill (backfill above drain naturally drained).

II - Design maximum flood, sudden drawdown in channel water level to bottom of the channel, backfill water level to Elevation 945.2 (tail-water surface).

III - Design maximum flood with sudden rise in reservoir, water level in channel to Elevation 945.2 msl (tail water), backfill submerged to Elevation of drains.

IIA - II with earthquake.

Section D-D of the wall on the upstream side and Section F-F on the downstream side at the north wall were analyzed for stability. One section of the south wall was also analyzed. Critical values of the factors of safety against sliding, location of resultant, and foundation pressures are shown in Table 2.

For Section D-D/49, North Wall, the resultant falls out of the middle third for Loading Case IIA (earthquake) but is within the base and is acceptable by the given criteria. Other stability requirements are satisifed.

For Section F-F/50, North Wall, the resultant falls out of the middle third for Loading Case II. The remaining stability requirements are satisfied for all of the loading cases and foundation pressures for all of the cases are within the allowable bearing pressure for the rock.

An anchorage system with deadmen, approximately 110 feet in length, is recommended as a remedial measure for the four channel wall monoliths downstream of the spillway (from Sta. 20+38 to Sta. 21+50). The anchorage system is to be designed so that the resultant will be located within the middle third of the wall. This system consists of horizontal tie rods, approximately 8 feet on center and located about 5 feet from the top of the wall, inserted through holes drilled in the wall and connected to a deadman anchor located approximately 35 feet back from the face of wall.

For Section D-D/49, South Wall, the resultant falls outside the middle third for Loading Case II but is within the base. An anchorage system, approximately 60 feet in length, is recommended for the three monoliths. The details for the anchorage system are similar to the system outlined for Section F-F/50, North Wall. Other stability requirements are satisfied.

The estimated cost for the two anchorage systems outlined is:

Section F-F/50, North Wall -	$19,000
Section D-D/49, South Wall -	$12,000
Total -	$31,000

TABLE 2

SPILLWAY WALLS

Wall Section	Loading Case	Location of Resultant: In Middle Third	Location of Resultant: In Base	Percent Base In Bearing	Resistance to Sliding Factor of Safety(*)	Bearing Pressures on Rock (Tons/S.F.): Maximum	Bearing Pressures on Rock (Tons/S.F.): Minimum
North Wall D-D/49	II	Yes	-	-	6.5	2.3	0.10
	III	Yes	-	-	9.8	1.5	0.55
	II-A	No	Yes	63	4.2	3.8	-
North Wall F-F/50	I	Yes	-	-	13.2	1.6	0.22
	II	No	Yes	90	9.6	1.7	-
	III	Yes	-	-	19.9	1.2	0.39
	II-A	No	Yes	57.7	6.9	2.7	-
South Wall D-D/49	II	No	Yes	96	9.5	1.7	-
	III	Yes	-	-	12.9	1.3	0.05
	IIA	No	Yes	56	6.5	2.9	-

*Factor of safety is for bond shear value of 40 psi and $\phi = 30^{\circ}$.

II - Section 5 - Stilling Basin Retaining Walls.

The stilling basin is constructed of reinforced concrete and is approximately 77 feet long. The bottom width varies from 7 feet at the conduit exit to 25 feet at the downstream end. The invert elevation of the 2-foot 6-inch slab drops from 903 ft. msl to 898 ft. msl, 35 feet from the portal. The 3-foot thick floor slab of the stilling basin is level and has a 3-foot high concrete end sill at the downstream end of the basin. The walls of the stilling basin are unreinforced concrete gravity sections above sound rock. The concrete lining below the gravity section is anchored to the rock by means of 1-inch round anchor bars. The lining is reinforced with 3/4-inch round reinforcement bars spaced 1 foot 0 inch center to center both ways. The base slab is anchored to the rock with 1-inch round anchors and reinforced with 3/4-inch round reinforcement bars spaced 1 foot 0 inch center to center both ways, top and bottom of slab. The height of the walls varies from 14 feet to 19 feet, measured above the channel floor. There are four monoliths, separated by contraction joints from 17 feet to 21 feet apart. Two unreinforced concrete gravity wing walls, above sound rock, are located on each side of the downstream end of the stilling basin. The east wall is 28 feet long with an average height of 10 feet. The west wall is 14 feet long with an average height of 5 feet. The west wall has a concrete lining below the gravity section similar to the stilling basin lining.

The stability analysis of the stilling basin was done in accordance with EM 1110-2-2400, Section 2-07f, and EM 1110-2-2502. The walls were designed for active pressure coefficients, modified where necessary for backfill slope. The wall sections were analyzed for the following loading cases:

I - Stilling basin empty, backfill submerged to drain.

II - Rapid closure of gates, water level inside at tail-water elevation, backfill submerged to elevation midway or higher between tail-water elevation before and after the reduction in flow.

IIA - II plus earthquake.

One section at the head wall, one section at the west wall, and three sections at the east wall were analyzed. Critical values of the factors of safety against sliding, location of resultant, and maximum foundation pressures are given in Table 3.

All wall sections analyzed, except for Section A/36, West Wall, have the resultant outside of the middle third for Loading Cases II and III with the percentage of the base in bearing varying between 87 and 99. The resultant for the head wall case, however, lies just outside the middle third and remedial work is not required. Maximum foundation pressures are within the allowable limits and the sections have adequate factor of safety against sliding for all the loading conditions.

An anchorage system with deadmen is recommended as a remedial measure behind all walls, except at the head wall. The total length of the anchorage system is approximately 200 feet. The anchorage is to be designed so that the resultant will be located within the middle third of the wall. This system consists of horizontal tie rods, approximately 8 feet on center and located about 5 feet from the top of wall, inserted through holes drilled in the wall and connected to a continuous deadman anchor located approximately 25 feet back from the face of the stilling basin walls.

The estimated cost for the anchorage system outlined above is $30,000.

TABLE 3

STILLING BASIN WALLS

Wall Section	Loading Case	Location of Resultant: In Middle Third	Location of Resultant: In Base	Percent Base In Bearing	Resistance to Sliding Factor of Safety(1)	Bearing Pressures on Rock (Tons/S.F.): Maximum	Bearing Pressures on Rock (Tons/S.F.): Minimum
East Wall D/36	I	Yes	-	-	13.5	1.4	0.07
	II	No	Yes	95	11.3	1.4	-
	II-A	No	Yes	48	8.3	2.8	-
East Wall C/36	I	Yes	-	-	13.9	1.4	0.70
	II	No	Yes	93	9.3	1.5	-
	II-A	No	Yes	35	6.5	4.0	-
East Wall B/36	II	Yes	-	-	13.2	1.1	0.6
	III	No	Yes	87	8.0	1.5	-
West Wall A/36	II	Yes	-	-	19.5	1.0	0.3
	II-A	No	Yes	81	13.2	1.5	-
Head Wall	II	No	Yes	99	21.9	0.8	-
	II-A	No	Yes	60	15.6	1.1	-

(1) With allowable bond shear of 40 psi and $\phi = 30^{\circ}$.

CONCLUSIONS

The intake structure, service bridge pier and abutment for MacDowell Dam satisfy, in all cases, the requirements of the new criteria for stability and no modifications or strengthening is required.

The remaining structures investigated satisfy this criteria; except for Loading Cases II and III, the resultant falls outside of the middle third in the majority of the sections investigated. The estimated cost and recommended remedial measures, consisting of tie rod anchorage or rock anchors, for these structures are as follows:

Spillway: Rock anchors at 8 feet 0 inch on center along crest	- $16,000
Spillway Walls: Tie rod anchorage, 170 feet of wall	- 31,000
Stilling Basin Retaining Walls: Tie rod anchorage, 200 feet of wall	- 30,000
TOTAL	- $77,000

MEMORANDUM

Site Visit to MacDowell Dam
West Peterborough, New Hampshire
February 1, 1974

The writer and Mr. Sanat Patwari were shown around by Mr. Rathburn, project manager. It was 15 degrees, cold, and windy. There was very little snow. We inspected visually and took several photographs of the following concrete structures:

1. Gate House. Water at Elevation 919 (15 feet above the sill elevation of 904). Concrete surfaces appear to be in good condition inside and outside. Railings freshly painted, everything clean.

2. Intake Channel. East side, top of one monolith has moved 3 inches at a construction joint; no more movement noticed within last seven years.

3. Bridge Pier and Abutment. Good condition, except that there is spalled concrete near one anchor bolt of bearing at the tower end.

4. Stilling Basin. Water rushing out, all stilling piers submerged. Walls in good condition.

5. Spillway Wall and Retaining Walls. About 4 miles away from the dam, shallow water on the reservoir side all frozen. West side of the north retaining wall shows about 2 inches of movement away from spillway at the joint. Visible concrete in good condition.

We did not notice any variances to conditions indicated on drawings and descriptions furnished to us that would affect the stability analysis of structures.

Jurgis Gimbutas

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CORPS OF ENGINEERS, NEW ENGLAND DIVISION
EDWARD MACDOWELL DAM
WEST PETERBOROUGH, NEW HAMPSHIRE

Stability Analysis of Structures

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